Contents

Chapter 8 Walls & Buried Structures

8.4 Retaining Walls

8.4.1 General

A retaining wall is a structure built to provide lateral support for a mass of earth or other material where a grade separation is required. Retaining walls depend either on their own weight, their own weight plus the additional weight of laterally supported material, or on a tieback system for their stability. Additional information is provided in Chapter 15 of the WSDOT Geotechnical Design Manual (GDM).

Standard designs for reinforced concrete cantilevered retaining walls, noise barrier walls (precast concrete, cast-in-place concrete, masonry, or timber), and geosynthetic walls are shown in the Standard Plans. The Region Design PE Offices are responsible for preparing the PS&E for retaining walls for which standard designs are available, in accordance with WSDOT Design Manual (DM) Section 1130.06. However, the Bridge and Structures Office may prepare PS&E for such standard type retaining walls if such retaining walls are directly related to other bridge structures being designed by the Bridge and Structures Office.

Structural earth wall (SEW) systems meeting established WSDOT design and performance criteria have been listed as “pre-approved” by the Bridge and Structures Office and the Materials Laboratory Geotechnical Division. The PS&E for “pre-approved” structural earth wall systems shall be coordinated by the Region Design PE Office with the Bridge and Structures Office, and the Materials Laboratory Geotechnical Division, in accordance with DM 1130.06.

The PS&E for minor non-structural retaining walls, such as rock walls, gravity block walls, and gabion walls, are prepared by the Region Design PE Offices in accordance with WSDOT DM 1130.06, and any other design input from the Region Materials Office or Geotechnical Division.

All other retaining walls not covered by the Standard Plans such as soil nail walls, soldier pile walls, soldier pile tieback walls and all walls beyond the scope of the designs tabulated in the Standard Plans, are designed in the Bridge and Structures Office according to the design parameters provided by the Materials Laboratory Geotechnical Division.

The Hydraulics Branch of the Design Office should be consulted for walls that subject to floodwater or are located in a flood plain. The State Bridge and Structures Architect should review the architectural features and visual impact of the walls during the Preliminary Design stage. The designer is also directed to DM 1130 and Chapter 15 of the GDM, which provide valuable information on the design of retaining walls.

8.4.2 Common Types of Walls

The majority of walls used by WSDOT are one of the following six types:

1. Proprietary Structural Earth (SE) Walls - Standard Specification Section 6-13.
2. Standard Reinforced Concrete Cantilevered Retaining Walls- Standard Plans D-1a to through D-1f and Standard Specification Section 6-11.
3. Soil Nail Walls - Standard Specification Section 6-15.
4. Soldier Pile Walls and Soldier Pile Tieback Walls - Standard Specification Sections 6-16 and 6-17.
5. Geosynthetic Walls (Temporary and Permanent) - Standard Plan D-3 and Standard Specification Section 6-14.
6. Noise Barrier Walls - Standard Plan D-2a through D-2y and Standard Specification Section 6-12.

Other wall systems, such as secant pile or cylinder pile walls, may be used based on the recommendation of the Geotechnical Division. These walls shall be designed in accordance with the current AASHTO LRFD Specifications.

A. Pre-approved Proprietary Walls

A wall specified to be supplied from a single source (patented, trademark, or copyright) is a proprietary wall. Walls are generally pre-approved for heights up to 33 ft. The Materials Laboratory Geotechnical Division will make the determination as to which pre-approved proprietary wall system is appropriate on a case-by-case basis. The following is a description of the most common types of proprietary walls:

1. Structural Earth Walls (SEW)

A structural earth wall is a flexible system consisting of concrete face panels or modular blocks that are held rigidly into place with reinforcing steel strips, steel mesh, welded wire, or geogrid extending into a select backfill mass. These walls will allow for some settlement and are best used for fill sections. The walls have two principal elements:

- Backfill or wall mass: a granular soil with good internal friction (i.e. gravel borrow).
- Facing: precast concrete panels, precast concrete blocks, or welded wire (with or without vegetation). At present, WSDOT has eight pre-approved proprietary wall systems as shown in the table in Appendix A. Design heights in excess of 33 feet shall be approved by the Materials Laboratory Geotechnical Division. If approval is granted, the designer shall contact the individual structural earth wall manufacturers for design of these walls before the project is bid so details can be included in the Plans. See sheet 8.1-A1.1 to A1.2 for details that need to be provided in the Plans for manufacturer designed walls. For additional information see the DM 1130 and Chapter 15 of the GDM. For the SEW shop drawing review procedure see Chapter 15 of the GDM.

2. Other Proprietary Walls

Other proprietary wall systems such as crib walls, bin walls, or precast cantilever walls, can offer cost reductions, reduce construction time, and provide special aesthetic features under certain project specific conditions.

For a list of these pre-approved proprietary walls and their height limitations, see Appendix A. The Region shall refer to DM 1130 for guidelines on the selection of wall types. The Materials Laboratory Geotechnical Division and the Bridge and Structures Office Preliminary Plans Unit must approve the concept prior to development of the PS&E.

B. Geosynthetic Wrapped Face Walls

Geosynthetic walls use geosynthetics for the soil reinforcement and part of the wall facing. Use of geosynthetic walls as permanent structures requires the placement of a cast-in-place or shotcrete facing. Details for construction are shown in Standard Plan D-3.

C. Standard Reinforced Concrete Cantilever Walls

Reinforced concrete cantilever walls consist of a base slab footing from which a vertical stem wall extends. These walls are suitable for heights up to 35 feet. Details for construction are given in the Standard Plans D-1a to D-1f. These Standard Plans were developed utilizing the AASHTO LFD specifications. Wall types 1 through 4 are suitable for use in Western Washington and designed for a seismic coefficient of 0.3. Wall types 5 and 6 are suitable for use only in Eastern Washington or where the seismic coefficient is less than 0.2. The maximum bearing pressure on the soil is tabulated on page DM 1130-30 for the 6 types of walls. The walls will be updated in the near future to meet the requirements of the AASHTO LRFD Specifications. For nonstandard designs; wall design shall be designed in accordance with Section 3 and Section 11 of the LRFD specifications. The major disadvantage of these walls is the low tolerance to post-construction settlement, which may require use of deep foundations (shafts or piling) to provide adequate support.

D. Soldier Pile Walls and Soldier Pile Tieback Walls

Soldier Pile Walls utilize wide flange steel members, such as W or HP shapes. The piles are usually spaced 6 to 10 feet apart. The main horizontal members are timber lagging designed to transfer the soil loads to the piles. For additional information see GDM Chapter 15.

See sheet 8.1-A3.1 to A3.5 for typical details.

E. Soil Nail Walls

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing steel bars called “nails” into a slope or excavation as construction proceeds from the “top down”. Soil nailing is a technique used to stabilize moving earth, such as a landslide, or as temporary shoring. Soil anchors are used along with the strength of the soil to provide stability. The Materials Laboratory Geotechnical Division designs the soil nail system whereas the Bridge and Structures Office designs the wall fascia. Presently, the FHWA Publication ‘Manual for Design & Construction Monitoring of Soil Nail Walls’ is being used for structural design of the fascia. See sheet 8.1-A2.1 to A2.5 for typical soil nail wall details.

F. Noise Barrier Walls

Noise barrier walls are primarily used in urban or residential areas to mitigate noise or to hide views of the roadway. Common types, as shown in the Standard Plans, include cast-in-place concrete panels (with or without traffic barrier), precast concrete panels (with or without traffic barrier), masonry blocks, and timber panels. The State Bridge and Structures Architect should be consulted for wall type selection. Design criteria for noise barrier walls are based on AASHTO’s *Guide Specifications for Structural Design of Sound Barriers*. Details of these walls are available in the Standard Plans D-2a to D-2y. DM 1140-3, figure 1140-1, tabulates the design wind speeds and various exposure conditions used to determine the appropriate wall type.

8.4.3 Design

A. General

All designs shall follow procedures as outlined in AASHTO LRFD Specifications Chapter 11, the GDM, and the Bridge Design Manual (BDM). The Materials Laboratory Geotechnical Division will provide the earth pressure diagrams and other geotechnical design requirements for special walls to be designed in the Bridge and Structures Office. Pertinent soil data will also be provided for pre-approved proprietary structural earth walls (SEW), non-standard reinforced concrete retaining walls, and geosynthetic walls.

B. Non-Standard Reinforced Concrete Retaining Walls

In general, concrete for reinforced concrete retaining walls shall be Class 4000 Concrete with a 28-day compressive strength of 4,000 psi. Typical load combinations and load factors can be found in AASHTO LRFD Figures C11.5.5-1 and C11.5.5-2. The following chart represents additional criteria for evaluating the overall stability of the wall.

$$\frac{(FS)P}{W} \leq 0.5$$

(P = total horizontal force on wall)
(W= total minimum vertical load)

For Standard Walls having a height (H) of 16 feet or less, the controlling load is the AASHTO LFD 10 kip collision load. This load occurs occasionally and will have a reduced factor of safety.

Wall Height, H Roadway Grade to Bottom of Footing	Overturning* M abt. toe resist M abt. toe loads	Location of Resultant*	Sliding FS(EP + Sur or 10k) < 0.5 Weight
H, 16 feet or less for 10K collision load	greater than 1.5	within middle 1/2 of footing	F.S. = 1.2
H, 17 feet or more for all Wall load cases	greater than2.0	within middle 1/2 of footing	F.S. = 1.5
Earthquake Group VII All Heights	greater than 1.5	Within middle 1/2 of footing	FS(EP + EQ) < 0.5 Weight FS = 1.1

*Both cases shall be met for determining wall stability for the service load condition.

Factor of Safety (FS) Table
Figure 8-1

The 10 kip collision load shall be distributed over 16 feet. This is the minimum wall length allowed for Type 2 Retaining Walls in the Standard Plans. In a special design, the AASHTO LRFD Extreme Event loading for vehicular collision must be analyzed. These loads are tabulated in LRFD Table A13.2-1. Although the current yield line analysis assumptions for this loading are not applicable to retaining walls, the transverse collision load (Ft) may be distributed over the longitudinal length (Lt) at the top of barrier. At this point, the load is distributed at a 45 degree angle into the wall. Future updates to the LRFD code will address this issue.

For sliding, the passive resistance in the front of the footing may be considered if the earth is more than 2 feet deep on the top of the footing and does not slope downward away from the wall. The design soil pressure at the toe of the footing shall not exceed the allowable soil bearing capacity supplied by the Materials Laboratory Geotechnical Division. For retaining walls supported by deep foundations (shafts or piles), refer to *Bridge Design Manual* Sections 9.5.1, 9.5.2, and 9.6 (see GDM).

C. Soldier Pile Tieback Walls

1. See Principles of Anchor Design - AASHTO LRFD Specification 11.9 "Anchored Walls." The Materials Laboratory Geotechnical Division will determine whether anchors can be used economically at a particular site based on the ability to install the anchors and develop anchor capacity. The presence of utilities or other underground facilities, and the ability to attain underground easement rights may also determine whether anchors can be installed.

   The anchor may consist of bars, wires, or strands. The choice of appropriate type is usually left to the Contractor but may be specified by the designer if special site conditions exist that preclude the use of certain anchor types. In general, strands and wires have advantages with respect to tensile strength, limited work areas, ease of transportation, and storage. However, bars are more easily protected against corrosion, and are easier to develop stress and transfer load.

   The geotechnical report will provide a reliable estimate of the nominal pullout resistances of the anchor, recommended anchor installation angles (typically 10° to 45°), no-load zone dimensions, and any other special requirements for wall stability for each project. AASHTO LRFD Section C11.9.5.1 outlines two procedures to determine the anchor design force. The capacity of each anchor shall be verified by testing. Testing shall be during the anchor installation (See Standard Specification Section 6-17.3(8) and GDM).

2. Corrosion Protection

   The Materials Laboratory Geotechnical Division will specify the appropriate protection system; the two primary types are:

   a. Simple Protection: The use of simple protection relies on Portland cement grout to protect the tendon, bar, or strand in the bond zone. The unbonded lengths are sheaths filled with anti-corrosion grease, heat shrink sleeves, and secondary grouting after stressing. Except for secondary grouting, the protection is usually in place prior to insertion of the anchor in the hole.

   b. Double Protection: a corrugated PVC, high-density polyethylene, or steel tube accomplishes complete encapsulation of the anchor tendon. The same provisions of protecting the unbonded length for simple protection are applied to those for double protection.

3. Determination of Tieback Spacing

   The preliminary anchor spacing can be determined from AASHTO LRFD C11.9.5.1. Typical pile spacing (horizontal) of 6 to 10 feet and anchor spacing (vertical) of 8 to 12 feet are commonly used. The minimum spacing of 4 feet in both directions is not recommended because it can cause a loss of effectiveness due to disturbance of the anchors during installation.

4. Design of Soldier Pile Tieback Walls

   a. Lateral Earth Pressures

      (1) Active pressure is assumed to act over one pile spacing above the base of excavation in front of the wall, and over the shaft diameter below the base of excavation in front of the wall. Passive pressure usually acts over three times the shaft diameter or pile spacing, whichever is smaller.

      (2) For permanent ground anchors, the anchor DESIGN LOAD, T, shall be according to AASHTO LRFD Specifications. For temporary ground anchors, the anchor DESIGN LOAD, T, may ignore extreme event load cases.

      (3) The lock-off load is 60 percent of the controlling factored design load for temporary and permanent walls (see GDM Chapter 15).

   b. Depth of Embedment

      For cantilever soldier piles without permanent ground anchors, the embedment should be determined to satisfy horizontal force equilibrium and moment equilibrium about the bottom of the soldier pile.

      For soldier piles with permanent ground anchors, the depth of embedment is determined by moment equilibrium of lateral force about the bottom of the soldier pile. Minimum embedment shall be 10 feet or as recommended by the Materials Laboratory Geotechnical Division.

   c. Design of Timber Lagging

      Most commonly, the lagging thickness is determined from past construction experience as related to depth of excavation, soil condition, and soldier pile spacing. In other cases the soil pressure distribution, as recommended by the Materials Laboratory Geotechnical Division, is used to determine the thickness of lagging.

A soil pressure distribution equal to 50 percent of the lateral earth pressure diagram is recommended for the design of simply supported lagging. The 50 percent reduction is due to the soil arching effect behind the wall. Neglecting the soil arching effect leads to unreasonably thick lagging for deep excavations with relatively larger soldier pile spacing. For design procedures see AASHTO LRFD 11.8.5.2.

d. Design of Concrete Fascia Panels

Concrete fascia panels shall be reinforced concrete and shall be designed according to the latest AASHTO LRFD Specifications. The minimum structural thickness of the concrete fascia panel shall be 9 inches. Architectural treatment of concrete fascia panels shall be indicated in the Plans.

Concrete strength shall not be less than 4,000 psi at 28 days. The wall is to extend 2 feet minimum below the finish ground line adjacent to the wall. Permanent drainage systems shall be provided to prevent hydrostatic pressures developing behind the wall. A cut that slopes toward the proposed wall will invariably encounter natural subsurface drainage.

Vertical chimney drains or prefabricated drainage mats can be used for normal situations to collect and transport drainage to a weep hole or pipe located at the base of the wall. Installing horizontal drains to intercept the flow at a distance well behind the wall may control concentrated areas of subsurface drainage (see GDM Chapter 15).

When concrete fascia panels are placed on soldier piles, the design earth pressure diagram and a generalized detail of lagging with a strongback (see sheet 8.1-A3.4) shall be shown in the Plans. This information will assist the Contractor in designing formwork that doesn't overstress the piles while concrete is being placed.

e. Design of Soldier Piles

The soldier piles shall be designed for shear, bending, and axial stresses according to the latest AASHTO LRFD and GDM design criteria. Due to the soil-structure interaction, a redistribution of lateral stresses is anticipated, resulting in a reduction of pressure near the center of spans between anchors and a concentration of pressure at supports.

8.4.4 Miscellaneous Items

A. Drainage

All reinforced concrete retaining walls shall have 3-inch diameter weepholes located 6 inches above final ground line and spaced about 12 feet apart. In case the vertical distance between the top of the footing and final ground line is greater than 10 feet, additional weepholes shall be provided 6 inches above the top of the footing. No weepholes are necessary in cantilever wingwalls. Drainage features shall be detailed in the Plans.

Weepholes can get clogged up or freeze up, and the water pressure behind the wall may start to increase. In order to keep the water pressure from building, it is important to have well draining gravel backfill and underdrains. Appropriate details must be shown in the Plans.

No underdrain pipe or gravel backfill for drains is necessary behind cantilever wingwalls. A 3 foot minimum thickness of gravel backfill shall be shown in the Plans behind the cantilever wingwalls. Backfill material shall be included with the civil quantities (not the bridge quantities). If it is necessary to excavate existing material for the backfill, then this excavation shall be a part of the bridge quantities for "Structure Excavation Class A Incl. Haul".

B. Joints

For cantilevered and gravity walls, joint spacing should be a maximum of 24 feet on centers. For counterfort walls, joint spacing should be a maximum of 32 feet on centers. For soldier pile and soldier pile tieback walls with concrete fascia panels, joint spacing should be 24 to 32 feet on centers. For precast units, the length of the unit depends on the height and weight of each unit. Odd panels for all types of walls shall normally be made up at the ends of the walls. Every joint in the wall shall provide for expansion. For cast-in-place construction, a minimum of 1/2 inch premolded filler should be specified in the joints. A compressible back-up strip of closed-cell foam polyethylene or butyl rubber with a sealant on the front face is used for precast concrete walls.

No joints other than construction joints shall be used in footings except at bridge abutments and where substructure changes such as spread footing to pile footing occur. In these cases, the footing shall be interrupted by a 1/2 inch premolded expansion joint through both the footing and the wall. The maximum spacing of construction joints in the footing shall be 120 feet. The footing construction joints should have a 6-inch minimum offset from the expansion joints in the wall.

C. Architectural Treatment

The type of surface treatment for retaining walls is decided on a job-to-job basis. Consult the State Bridge and Structures Architect during preliminary plan preparation. The wall should blend in with its surroundings and complement other structures in the vicinity. The tops of walls are usually smooth flowing curves in elevation view. See Retaining Wall Standard Sheets for top of wall and ground line relationship and also for cambering of front of cantilevered retaining walls.

D. Shaft Backfill for Soldier Pile Walls

1. Soldier Pile Walls without Permanent Ground Anchors

   For this application, controlled density fill (CDF, 145 pcf) should be used for the drilled portion of the pile. In this situation, the fill only needs to be strong enough to transfer active and passive loads between the soil and the soldier pile.

2. Soldier Pile Walls With Permanent Ground Anchors

   For this application, structural concrete (Concrete Class 4000P) should be used below final grade (below the cut line in front of the soldier pile wall), and CDF should be used above the bottom of the timber lagging. Concrete Class 4000P can be used in a wet hole application. Structural concrete is needed below final grade to assure vertical load transfer from the soldier pile shaft into the supporting soil. The load is applied to the soldier pile from the vertical component of the sloping permanent ground anchors.

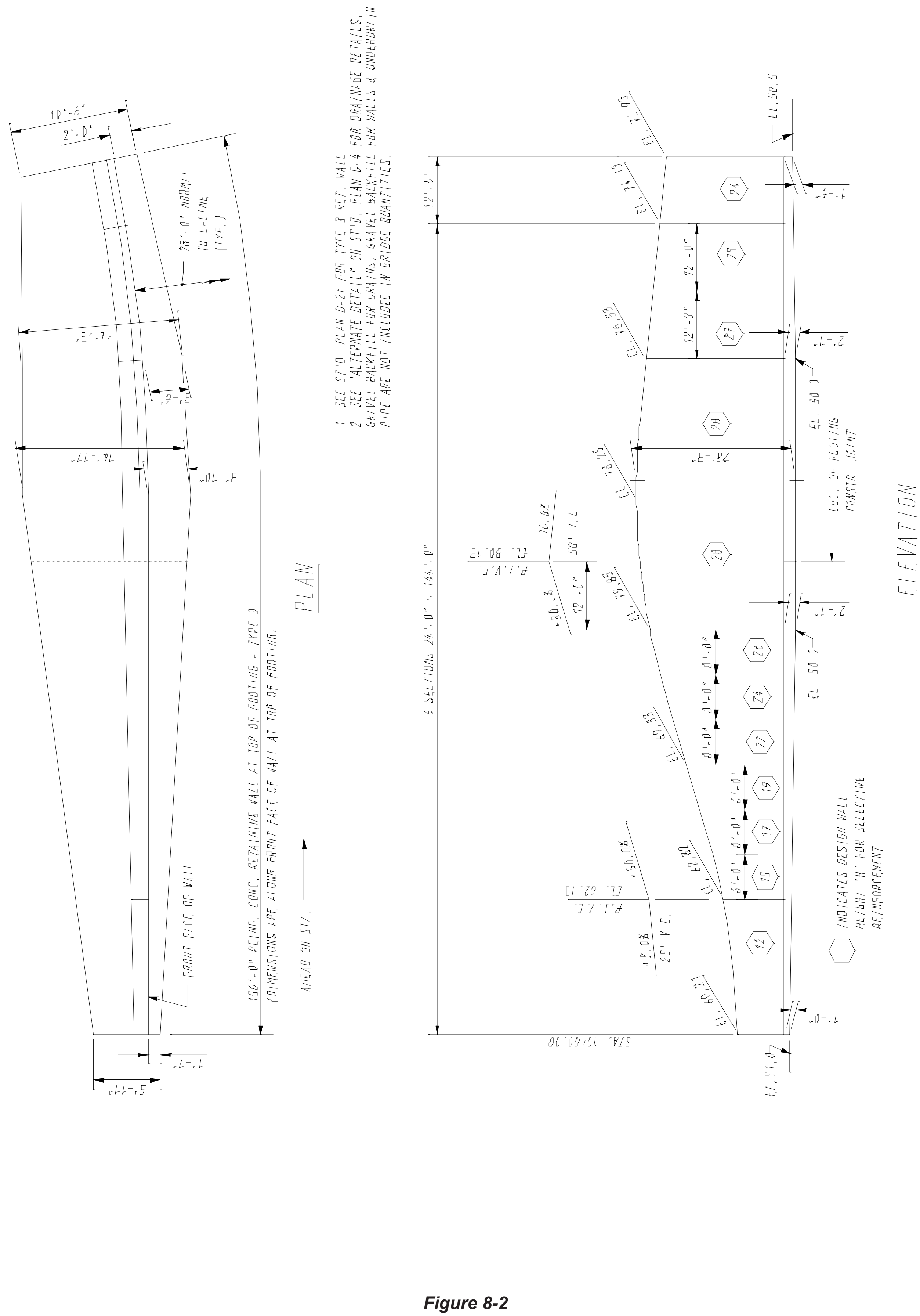
1. SEE ST'D. PLAN D-21 FOR TYPE 3 RET. WALL.
2. SEE "ALTERNATE DETAIL" ON ST'D. PLAN D-4 FOR DRAINAGE DETAILS, GRAVEL BACKFILL FOR DRAINS, GRAVEL BACKFILL FOR WALLS & UNDERDRAIN PIPE ARE NOT INCLUDED IN BRIDGE QUANTITIES.
PLAN
156'-0" REINF. CONC. RETAINING WALL AT TOP OF FOOTING - TYPE 3
(DIMENSIONS ARE ALONG FRONT FACE OF WALL AT TOP OF FOOTING)
AHEAD ON STA.
FRONT FACE OF WALL
ELEVATION
6 SECTIONS 24'-0" = 144'-0"
LOC. OF FOOTING CONSTR. JOINT
STA. 10+00.00
INDICATES DESIGN WALL HEIGHT "H" FOR SELECTING REINFORCEMENT

Figure 8-2

E. Detailing of Standard Reinforced Concrete Retaining Walls

1. In general, the “H” dimension shown in the retaining wall Plans should be in foot increments. Use the actual design “H” reduced to the next lower even foot for dimensions up to 3 inches higher than the even foot.

   Examples: Actual height = 15′-3”↑, show “H” = 15’ on design plans

   Actual height > 15′-3”↑, show “H” = 16’ on design plans

   For walls that are not of a uniform height, “H” should be shown for each segment of the wall between expansion joints or at some other convenient location. On walls with a steep slope or vertical curve, it may be desirable to show 2 or 3 different “H” dimensions within a particular segment. The horizontal distance should be shown between changes in the “H” dimensions.

   The value for “H” shall be shown in a block in the center of the panel or segment. See Example, Figure 9.4.4-1.

2. Follow the example format shown in Figure 8.-2.
3. Calculate approximate quantities using the Standard Plans.
4. Wall dimensions shall be determined by the designer using the Standard Plans.
5. Do not show any details given in the Standard Plans.
6. Specify in the Plans all deviations from the Standard Plans.
7. Do not detail reinforcing steel, unless it deviates from the Standard Plans.

8. For pile footings, use the example format with revised footing sizes, detail any additional steel, and show pile locations. Similar plan details are required for footings supported by shafts.

8.5 Miscellaneous Underground Structures

8.5.1 General

Miscellaneous underground structures consist of box culverts, precast reinforced concrete three-sided structures, detention vaults, metal pipe arches, and tunnels. These structures are designed in accordance with the AASHTO Standard Specifications for Highway Bridges, and applicable ACI design and ASTM material specifications.

Precast reinforced concrete three sided structures, constructed in accordance with the current WSDOT General Special Provisions (GSP's) for these structures, shall be designed under the Load Factor Design (LFD) method of the AASHTO Standard Specifications For Highway Bridges. Detention vaults shall also be designed in accordance with the criteria in the ACI 350 Code Requirements for Environmental Engineering Concrete Structures. The detention vault design should satisfy the most stringent criteria of the LFD design method of AASHTO Standard Specifications for Highway Bridges and that of ACI 350. Requirements more stringent than either of these two codes may be desirable for unusual structures.

Generally, seismic design criteria does not control the design of underground structures unless the peak ground acceleration exceeds 0.3g, where g is the acceleration due to gravity. This is because the structures are supported on all sides by soil and rock, and move as a unit with the adjacent soil. See Reference by Miller and Constantino (1994). As with any structure, a geotechnical soils report with loading or pressure diagrams, settlement criteria, and ground water levels will be needed from the Materials Laboratory Geotechnical Services Branch in order to complete the design.

8.5.2 Design

A. Box Culverts

Box culverts are four-sided rigid frame structures and are either made from cast-in-place (CIP) reinforced concrete or precast concrete. In the past, standardized box culvert plan details were shown in the WSDOT Standard Plans, under the responsibility of the HQ Hydraulics Office. These former Standard Plans have been deleted and are no longer available. Now box culvert design is standardized under applicable AASHTO material specifications, and design plans are not required in the PS&E. Box culverts with less than two feet of cover shall be in accordance with AASHTO M 273. Box culverts with two feet or more of cover shall be in accordance with AASHTO M 259.

B. Precast Reinforced Concrete Three-Sided Structures

Precast reinforced concrete three-sided structures are patented or trademarked rigid frame structures made from precast concrete. Some fabricators of these systems are: Utility Vault Company, Central Pre-Mix Prestress Company, and Bridge Tek, LLC. These systems require a CIP concrete or precast footing that must provide sufficient resistance to the horizontal reaction or thrust at the base of the vertical legs.

The precast concrete fabricators are responsible for the structure design and the preparation of shop plans. The fabricators of systems which have received WSDOT pre-approval are specified in the GSP's. The bridge designer reviewing the project will be responsible for reviewing the fabricator's design calculations and details with consultation with the Construction Support Unit. Under the current GSP, precast reinforced concrete three sided structures are limited to spans of 26 feet or less. However, in special cases it may be necessary to allow longer spans, with the specific approval of the Bridge and Structures Office. Several manufacturers advertise spans over 40 feet.

C. Detention Vaults

Detention vaults are used for stormwater storage and are to be watertight. These structures can be open at the top like a swimming pool, or completely enclosed and buried below ground. Two references for tank design are the PCA publications *Rectangular Concrete Tanks*, Revised 5th Edition (1998) and *Design of Liquid-Containing Structures for Earthquake Forces* (2002).

Buoyant forces from high ground water conditions should be investigated for permanent as well as construction load cases so the vault does not float. Controlling loading conditions may include: backfilling an empty vault, filling the vault with stormwater before it is backfilled, or seasonal maintenance that requires draining the vault. The designer should check with the Region Project Engineer's Office to see if weep holes can be placed at or above the high groundwater level to relieve external hydrostatic pressure should buoyant forces be too large to resist.

ACI 350 requires Environmental Durability Factors in addition to the larger load factors of 1.4 for dead load and 1.7 for fluid pressure (ACI 350 controls over AASHTO). These factors are:

1.3 for flexure

1.65 for direct tension

1.3 for shear beyond that of the capacity provided by the concrete

The LFD strength equations become:

1. Flexural Reinforcement

   $M_n > 1.3(1.4M_{DL} + 1.7M_{EP} + 1.7\ M_{LL} + 1.7M_{FL})$

2. Direct Tension Reinforcement

   $F_n > 1.65(1.4F_{DL} + 1.7F_{EP} + 1.7\ F_{LL} + 1.7F_{FL})$

3. Stirrup Reinforcement

   The designer should increase the wall or slab thicknesses so that shear stirrups are not required.

   $\phi V_S > 1.3(V_U - \phi V_C)$

4. Concrete Shear and Compression

   $F_n > 1.3(1.4F_{DL} + 1.7F_{EP} + 1.7\ F_{LL} + 1.7F_{FL})$

ACI 350 has stricter criteria for cover, crack control, and spacing of joints than the AASHTO Specifications. Cover is not to be less than 2 inches, no metal or other material is to be within 1-1/2 inches from the formed surface, and the maximum bar spacing shall not exceed 12 inches. Crack control criteria is stricter with "z" not exceeding 115 kips per inch under normal exposure conditions; this corresponds to a crack width of 0.010 inches.

Joints in the vault's top slab, bottom slab and walls shall allow dissipation of temperature and shrinkage stresses, thereby reducing cracking. The amount of temperature and shrinkage reinforcement is a function of reinforcing steel grade " and length between joints (See Table 7.12.2-1 ACI 350). All joints shall have a shear key and a continuous and integral PVC waterstop with a 4-inch minimum width. The purpose of the waterstop is to prevent water infiltration and exfiltration.

For walls in contact with liquid that are over 10' in height, the minimum wall thickness is 12". This minimum thickness is generally good practice for all external walls, regardless of height, to allow for 2 inches of cover as well as space for concrete placement and vibration.

After the forms are placed, the void left from the form ties shall be coned shaped, at least 1 inch in diameter and 1 1/2 inches deep, to allow proper patching.

Detention vaults shall not be located within the roadway prism.

Original signed plans of all closed top detention vaults with access shall be forwarded to the Bridge Plans Engineer in the Bridge Project Unit (see Bridge Design Manual Section 12.4.10.B). This ensures that the Bridge Preservation Office will have the necessary inventory information for inspection requirements.

D. Metal Pipe Arches

Pipe arch systems are similar to precast reinforced concrete three sided structures in that these are generally proprietary systems provided by several manufacturers, and that their design includes interaction with the surrounding soil. Pipe arch systems shall be designed in accordance with Section 12 of the AASHTO Standard Specifications for Highway Bridges.

E. Tunnels

Tunnels are unique structures in that the surrounding ground material is the structural material that carries most of the ground load. Therefore, geology has even more importance in tunnel construction than with above ground bridge structures. In short, geotechnical site investigation is the most important process in planning, design and construction of a tunnel.

Tunnels are not a conventional structure, and estimation of costs is more variable as size and length increase. Ventilation, safety access, fire suppression facilities, warning signs, lighting, drainage, operation and maintenance are extremely critical issues associated with the design of tunnels and will require the expertise of geologists, tunnel experts and mechanical engineers.

For motor vehicle fire protection, a standard has been produced by the National Fire Protection Association. This document, "NFPA 502 – Standard for Road Tunnels, Bridges, and Other Limited Access Highways", uses tunnel length to dictate minimum fire protection requirements:

300 feet or less: no fire protection requirements

300 to 800 feet: minor fire protection requirements

800 feet or more: major fire protection requirements

Some recent WSDOT tunnel projects are:

I-90 Mt. Baker Ridge Tunnel Bore Contract: 3105 Bridge No: 90/24N

This 1500 foot long tunnel is part of the major improvement of Interstate 90. Work was started in 1983 and completed in 1988. The net interior diameter of the bored portion, which is sized for vehicular traffic on two levels with a bike/pedestrian corridor on the third level, is 63.5 feet. The project is the world's largest diameter tunnel in soft ground, which is predominantly stiff clay. Construction by a stacked-drift method resulted in minimal distortion of the liner and insignificant disturbance at the ground surface above.

Jct I-5 SR 526 E-N Tunnel Ramp Contract: 4372 Bridge No: 526/22E-N

This 465 foot long tunnel, an example of the cut and cover method, was constructed in 1995. The interior dimensions were sized for a 25 foot wide one lane ramp roadway with a vertical height of 18 feet. The tunnel was constructed in 3 stages. 3 and 4 foot diameter shafts for the walls were placed first, a 2 foot thick cast-in-place top slab was placed second, then the tunnel was excavated, lined and finished.

I-5 Sleater-Kinney Bike/Ped. Tunnel Contract: 6031 Bridge No: 5/335P

This 122 foot long bike and pedestrian tunnel was constructed in 2002 to link an existing path along I-5 under busy Sleater-Kinney Road. The project consisted of precast prestressed slab units and soldier pile walls. Construction was staged to minimize traffic disruptions.

8.5.3 References

1. AASHTO LRFD Bridge Design Specifications, 2nd Ed., w/Interims
2. AASHTO Standard Specifications for Highway Bridges, 17th Ed., 2002
3. ACI 350R-01 Code Requirements for Environmental Engineering Concrete Structures, ACI
4. ACI 318-95 Building code Requirements for Structural Concrete, ACI, 1995.
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